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Thermo-poro-mechanical analysis of landslides: from creeping behaviour to catastrophic failure

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19 Catastrophic landslides are characterized by high velocities (1 to 100 m/s), large displacements
20 (tens to hundreds of meters) and, in most cases, by a large mobilized mass, in excess of 1Mm^3 .

21 The scope of the paper encompasses planar and compound sliding motions which may exhibit
22 creeping behaviour during a certain period but may evolve to a very rapid motion. Thermo-
23 mechanical interactions, at the scale of the sliding surface, are accepted as a critical aspect to
24 explain these motion phases and their relationship.

25 The paper reviews the mechanisms leading to strength reduction of the failure surface and the
26 published thermo-mechanical formulations. They share a common basic structure. The sliding
27 kinetics and global equilibrium are described at a large scale (the landslide itself) and the
28 evolving shearing strength at the sliding surface derives from the “local” analysis of the
29 shearing band and its vicinity. Pore pressures, temperatures and related variables are
30 estimated by resolving a set of balance equations. Both scales are fully coupled. A significant
31 aspect analyzed in detail in the paper is the transition from creeping motions to a rapid event.
32 It requires the joint consideration of strain-rate effects on friction and the thermo-poro-
33 mechanical analysis of the shearing band and its vicinity. Results are found in terms of
34 dimensionless numbers which control the entire phenomenon. Calculation of the slide
35 evolution requires special numerical techniques described in the paper. Band permeability is
36 found to be the dominant property controlling the triggering of fast motions. The creeping
37 stage and the eventual slide blowup are intimately linked. This relationship is explored in the
38 paper.

39 Although this is not pursued in this paper, the models presented can be readily used to back-
40 analyse relevant case histories or, in principle, even to carry out predictive modelling, provided
41 an adequate calibration is available for the material parameters.

42 KEYWORDS: landslides, shear strength, temperature effects, strain localization, creep, pore
43 pressures.

44

45 **Notation list**

- 46 A : model parameter defining the effect of velocity on strength
- 47 a : model parameter defining the effect of strain rate on friction angle
- 48 c_s : specific heat of solid particles
- 49 c_w : specific heat of water
- 50 D : planar landslide thickness
- 51 E : energy barrier
- 52 e : thickness of the shear band
- 53 E_1 : reference energy barrier
- 54 f^* : basic friction for $v=v^*$ and $\psi=0$
- 55 f_v : function defining the effect of sliding velocity on friction angle
- 56 g : gravitational acceleration
- 57 \hat{H} : dimensionless heat generated in the shear band
- 58 h_w : water height above the sliding plane
- 59 k : saturated permeability
- 60 L_{u_w} : z coordinate where a boundary condition for excess pore water pressure is applied
- 61 L_θ : z coordinate where a boundary condition for temperature is applied
- 62 \hat{L}_θ : dimensionless L_θ
- 63 m_{soil} : compressibility coefficient of the soil skeleton

- 64 m_v : oedometric compressibility coefficient of soil
- 65 n : porosity
- 66 p_w : initial pore water pressure
- 67 \hat{p}_w : dimensionless initial pore water pressure
- 68 R : universal gas constant
- 69 SF : safety factor
- 70 T : absolute temperature
- 71 t : time
- 72 u_w : excess pore water pressure
- 73 \hat{u}_w : dimensionless excess pore water pressure
- 74 \hat{u}_w^{\max} : dimensionless maximum excess pore water pressure
- 75 v^* : velocity below which friction does not depend on velocity
- 76 v : sliding velocity
- 77 \hat{v} : dimensionless sliding velocity
- 78 v_1 : reference shearing rate
- 79 v_{ref} : reference velocity
- 80 \hat{v}_{ref} : dimensionless reference velocity
- 81 z : vertical spatial coordinate
- 82 \hat{z} : dimensionless vertical spatial coordinate

83	α_w :	compressibility coefficient of water
84	β :	inclination of sliding surface
85	β_s :	thermal expansion coefficient of solid particles
86	β_{soil} :	thermal expansion coefficient of saturated porous media
87	β_w :	thermal expansion coefficient of water
88	$\dot{\delta}$:	sliding velocity
89	$\dot{\varepsilon}$:	strain rate
90	$\dot{\varepsilon}_0$:	reference strain rate
91	ϕ'_0 :	reference frictional angle associated with a reference strain rate ($\dot{\varepsilon}_0$)
92	ϕ' :	effective friction angle
93	γ_w :	water specific weight
94	Γ :	Fourier's thermal conductivity of saturated porous media
95	Γ_s :	Fourier's thermal conductivity of solid particles
96	Γ_w :	Fourier's thermal conductivity of water
97	μ :	friction coefficient
98	μ_0 :	reference friction coefficient
99	Θ :	dimensionless coefficient associated with heat flow in the heat balance equation
100	θ :	temperature
101	$\hat{\theta}$:	dimensionless temperature

102	θ_0 :	reference temperature
103	Π :	dimensionless coefficient associated with thermal expansion of porous rock in the
104		mass balance equation of water
105	ρ :	density of saturated porous media
106	ρ_s :	density of solid particles
107	ρ_w :	water density
108	$(\rho c)_s$:	specific heat of saturated porous media
109	σ :	normal stress to the sliding plane
110	σ_c :	normal stress to the sliding plane acting at molecular scale
111	τ :	shear stress along the sliding plane
112	τ_c :	shear adhesion stress acting at molecular scale
113	Σ :	dimensionless coefficient associated with stress induced volumetric deformation in the
114		mass balance equation of water
115	ξ_0 :	reference value of ξ
116	ξ :	state variable which accounts for strengthening and weakening effects on the friction
117		coefficient
118	Ω :	active volume of contact bonds
119	Ψ :	dimensionless coefficient associated with the source term in the heat balance
120		equation

121 $\bar{\Psi}$: variable to include other effects than those associated with shearing velocity on
122 frictional strength

123

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1. INTRODUCTION

Large catastrophic landslides are a constant threat to human communities and infrastructure works. Their danger derives from the large volumes of rock mass they involve (well in excess of 1million m³) and their high estimated sliding velocities (10m/s to 100m/s). Sosio et al (2008) list a number of historical rock avalanches and their main characteristics. The known number of catastrophic landslides is, however, much larger and there is a continuous arrival of new cases, triggered by different natural and anthropogenic causes: earthquakes, heavy rain, reservoir impounding and rapid drawdown and excavations or river erosion. A wider view of catastrophic landslides at a regional scale and its interaction with other geodynamic processes is given by Hewitt et al (2008). Active mountain formation and the associated ongoing crustal plate collisions indicate the natural occurrence of these events, which is not expected to decrease in time.

A classic landslide that has been extensively studied is that of Vaiont; Hendron and Patton (1985), Müller (1964), Nonveiller (1987), Semenza (2001) and Paronuzzi et al (2013) provide detailed and informative contributions. Other large scale catastrophic landslides associated with reservoir operations have been reported by Alcántara-Ayala and Domínguez-Morales (2008) (San Juan de Grijalva slide in Mexico, 2007); Wang et al (2004) and Dai et al (2004) (Qiangjianping slide in China, towards the reservoir of the Three Gorges Project, 2003); Chamot (1993), Plaza-Netos and Zevallos (1994), Schuster *et al.*, (2002) and Harden (2004) (La Josefina landslide, Ecuador, in 1993). An often reported case in recent years is the giant Tsaoling landslide, Taiwan, that was triggered by the 1999 Chi-Chi earthquake (Chen et al, 2003; Tang et al, 2009; Liao et al, 2011; Wu and Chen, 2011; Yang et al, 2014.) In general, the

149 interpretation of earthquake induced motion requires velocity- and, in some cases,
150 displacement-dependent shear strength weakening of the failure surface. This “slip
151 weakening” effect is discussed below because of its attributed relevance to the sudden
152 acceleration of rapid slides.

153 The morphology and dynamics of these events is quite diverse. Hungr et al (2014) provide an
154 updated classification of landslides but, for the purpose of this paper, we may broadly
155 distinguish flow-like motions from motions with a dominant sliding mode of deformation. This
156 distinction is relevant when considering the mathematical methods of analysis available. In the
157 first case hydrodynamic motion equations and soil/rock properties often associated with the
158 concept of viscosity have been developed (Iverson et al 1997; Cascini et al 2010; Pastor et al
159 2014). In the second case attention is concentrated on the position and frictional
160 characteristics of the sliding surfaces, which should satisfy conditions of kinematic
161 compatibility. This is the approach followed here. Further, a large number of numerical
162 modelling techniques exist, capable of simulating landslide initiation and motion. Methods for
163 the analysis of continua such as the FEM or the MPM (Duncan 1996; Więckowski et al 1999;
164 Darve and Laouafa, 2000; Bardenhagen & Kober 2004; Conte et al 2010; Andersen & Andersen,
165 2010; Pinyol et al 2011; Zabala and Alonso 2011; Yerro et al 2014; Alonso et al 2014) may
166 describe the deforming mass by elastoplastic constitutive equations familiar in Soil Mechanics.
167 DEM and related procedures describe the moving mass by laws of interaction between
168 particles or blocks (Cleary and Campbell 1993; Campbell et al 1995).

169 If the main deformation mechanism considered is one of sliding along localized and thin
170 shearing surfaces, the underlying assumption is that these surfaces are already fully
171 developed. Of the four different types of slope movements described by Leroueil (2001) we
172 are thus concerned with the post-failure stage following first-time failures, which includes the
173 movement of the sliding mass from immediately after the full development of the shearing

surface till it comes to rest, and with reactivated slides, where movement restarts along a pre-existing failure surface developed during a past event. It is physical and physicochemical processes occurring at these shearing surfaces that determine the slide's motion and its evolution in time. However, the enormous difference of scale between the thickness of the shearing band and that of the slide raises difficulties. Integration of both scales in a common calculation procedure requires a trade-off, as methods providing a reasonable description of the entire moving mass are ill-conditioned to describe complex physical mechanisms at the scale of a shearing band. However, a combination of simple dynamic equilibrium models for the entire slide and closer attention to the deforming shearing bands proves useful. This is the approach followed here, in line with Voight and Faust (1982), Hendron and Patton (1985), Vardoulakis (2002), Veveakis et al (2007), Pinyol and Alonso (2010a, 2010b), Goren and Aharonov (2009), Cecinato et al (2011), Cecinato and Zervos (2012). In all these cases the heat generated by friction and the resulting pressurization of interstitial water were fundamental parts of the analysis; their relevance in explaining high landslide velocities was first highlighted in the pioneering contributions of Habib (1975); Uriel and Molina (1977) and Voigt and Faust, (1982). Table 1 summarizes the main aspects of published contributions on thermal effects and rapid landslide triggering.

The scope of the paper is limited to landslides whose evolving geometry is characterized by soil or rock masses that essentially maintain their original bulk stiffness and strength. The motion is explained by localized shearing surfaces such as those often located in clayey strata described as indurated claystones or overconsolidated clays. Interestingly, Tang et al (2009) observe that, despite the heavy internal fracturing observed in many large landslides, they show a "remarkable tendency to remain in a more or less unchanged sequential order"; this helps extend the analysis described here to a larger class of landslides.

From a geotechnical perspective, it is important to distinguish between first-time failures and reactivation of ancient landslides. The first type develops in “intact” sites. They are difficult to analyze, especially when brittle materials are involved in the vicinity of the potential failure surface, as is the case of hard soils or soft argillaceous rocks, in particular those of high plasticity. The strength operating in practice and the geometry of the failure surface are difficult to predict because the failure mechanism develops progressively. Early classic studies on this topic were published by Skempton et al. (1967), Bjerrum (1967) and Bishop (1967, 1971). More recent contributions include those by Cooper (1998), Potts *et al.* (1990), Dounias *et al.* (1990), Mesri and Shahien (2003), Gens and Alonso (2006) and Zabala and Alonso (2010).

In contrast, re-activated ancient landslides occur on a pre-existing sliding surface that has been subjected to an increasing history of accumulated relative displacements. Hence, it is expected that cohesion will be insignificant on the sliding surface and the friction angle will be close or equal to residual values. It is widely accepted that ancient landslides exhibit a low safety factor close to conditions of strict equilibrium ($SF=1$). ICOLD (2002) reports that in at least 75% of cases where old landslides (active or inactive) are disturbed, e.g. by an excavation or by submerging the toe, slide reactivation or an increase in velocity is observed. It is also frequently observed, especially in consolidated clayey strata, that a very low value of friction angle, even lower than the residual friction determined in ring shearing tests, operates in joints and shearing zones of ancient landslides. The evolution of residual strength during resting times is a controversial subject which has been discussed in Alonso and Pinyol (2014).

Consider, as an introduction to the remainder of the paper, the motion of a block sliding on a friction-resistant surface inclined at angle β . Velocity increases linearly with time, t , following the equation:

$$v = g \sin \beta (1 - SF) t \quad (1)$$

where g is the gravity acceleration and SF the initial safety factor, defined as the ratio between resisting (shear strength) and disturbing shear stress. If $SF=1.0$ the block remains still. To initiate motion it is necessary to introduce some imbalance between resisting and disturbing stress. In real situations involving re-activated slides the imbalance may have different origins but it is typically small; e.g. raising the water level in a reservoir slowly affects pore pressures in the valley upstream.

For $SF=0.99$ and $\beta=10^\circ$, equation (1) predicts that a catastrophic sliding velocity of 1m/s will be reached in 58s. However field evidence does not support this result. A reasonable explanation is that the friction angle increases slightly as slide motion causes the rate of shearing on the failure plane to increase. Then SF increases to $SF=1$ and the slide reaches dynamic equilibrium and exhibits creeping motion, i.e. it moves with constant or near-constant velocity that is also relatively low.

Experience also indicates (this is the case of Vaiont) that a relatively slow creep motion of a few mm/day may evolve to 30m/s in 15s. However equation (1) predicts, for the same slope angle, that for a small reduction of SF by 1% it takes 30 minutes to reach 30m/s. Therefore equation (1) is unable to explain observations at the two scales of the motion of interest in practice: the creeping states and the eventual catastrophic outcome.

Material brittleness may explain a significant reduction of shear strength at the start of the motion. It may be invoked in cases of first time failures but it cannot be easily justified in re-activated slides or in creeping motions that may evolve into a rapid slide. It may be also present in compound sliding motions which require shearing across “intact” rock and not only on the basal sliding surface. This is certainly an acceptable scenario in some cases but it cannot explain the rapid motion of planar slides, which is a common case.

This paper is organized as follows: The question of the strain rate dependence of friction is discussed in Section 2. In Section 3 a thermo-poro-mechanical model for an infinite planar slide is presented, using appropriate dimensionless variables. Section 4 discusses the numerical methods developed to solve the corresponding initial/boundary value problem, and Section 5 contains a range of numerical results that provide insights into the underlying phenomena. Some conclusions are presented in Section 6.

2. STRAIN RATE DEPENDENT FRICTION AND PRECURSORY SLIDING MOTIONS

Before discussing some results of shearing experiments it is convenient to briefly examine some basic concepts of the theoretical understanding of friction. The initial contributions are presented in Bowden & Tabor (1964). Mitchell (1976) and Rice (2001) describe how employing the theory of “activation energy” at the level of microscopic contacts leads to the common law of friction. A frictional contact is idealised as a number of isolated contact points (Fig. 1) where adhesion (τ_c) and normal stresses (σ_c) are understood to act at the scale of molecular interactions between the minerals in contact. Normal and shear stresses are expected to reach high local values and are controlled by chemical reactions. Using equilibrium, the macroscopic shear and normal stresses (τ and σ) are related to their microscopic counterparts as:

$$\tau = \sigma \frac{\tau_c}{\sigma_c} \quad (2)$$

which justifies why shear stress is linearly related to normal stress.

However, under transient shearing, chemical reactions at the molecular scale of the isolated process zones of the local contacts are expected to change the value of the adhesion stresses (τ_c). If local chemical reactions follow a rate process, their velocity of reaction can be written:

$$v = v_1 \exp\left(-\frac{E}{RT}\right) \quad (3)$$

where E is the activation energy, T is absolute temperature, R is the gas constant and v_1 is a reference shearing rate. The interpretation of this equation is that the energy E in equation (3) allows a shearing rate velocity v . E has the meaning of a threshold energy barrier: if it decreases, velocity increases. Applied stresses decrease the energy barrier and increase the velocity of the processes taking place at the point contacts and, therefore, the velocity of deformation. Stress is understood as energy per unit volume, so the effect of stress on the energy barrier E can be expressed as:

$$E = E_1 - \tau_c \Omega \quad (4)$$

where Ω is an active volume of contact bonds and E_1 is a reference energy barrier. Equations (3) and (4) lead to

$$\tau = \sigma \left[\frac{E_1}{\Omega \sigma_c} + \frac{RT}{\Omega \sigma_c} \ln \frac{v}{v_1} \right] \quad (5)$$

which predicts a linear increase of friction with the logarithm of shearing velocity. The reference term $E_1/(\Omega \sigma_c)$ is interpreted as a basic friction which may depend on the state of the frictional surface; “state” may include shearing relative displacement, applied stress and time (Rice et al, 2001). The friction law may then be written,

$$\frac{\tau}{\sigma} = f = f^* + \bar{\psi} + A \ln \frac{v}{v^*} \quad (6)$$

where f^* is a basic friction and $\bar{\psi}$ includes effects other than those associated with shearing velocity.

Experimental results at low shearing rates tend to support equation (6). For instance, based on direct and ring shear tests of highly plastic clay layers of the Clearwater formation in Alberta, Canada, Wedage et al. (1998) proposes the following empirical relationship between residual friction and shearing rate:

$$\tan \phi = \tan \phi_0 \left(1 + a \ln \frac{\dot{\epsilon}}{\dot{\epsilon}_0} \right) \quad (7)$$

Cooper et al (1999) investigated the effect of rate of shear on ring shear tests performed on high plasticity Gault clay. Shearing rates varied almost four orders of magnitude, from 0.5 mm/min to 0.0001 mm/min, but still within the range expected in creeping slide motions. Tika et al (1996) extended the rate of shearing to the range 10^{-5} to 10^4 mm/min and concluded that the evolution of residual strength depends on the type of soil. They found that some soils exhibit a “negative” rate effect of the residual strength, which was explained as result of an increase in water content in the shear zone, because of the dilatant effect of “turbulent” shearing (which requires some granular content). Skempton (1985) presented results of ring shear test on clays conducted at rates from 100 times faster to 100 times slower than the commonly used 0.01mm/min; they show that the residual strength increased by about 2.5% per log cycle increase in strain rate, an increase that Skempton (1985) acknowledges can cause large changes in the rate of movement.

Direct shear tests on polished surfaces of rock (Dieterich, 1979, Ruina, 1983) support a linear increase of the friction coefficient with the logarithm of the shearing rate. Alternative proposals have also been presented, e.g. Davis *et al.* (1993) propose:

$$\frac{\tau}{\sigma} = f = f^* + b \frac{v}{v_r} \exp \left(\frac{-v}{v_r} \right) \quad (8)$$

The velocity term in Equation (8) provides an initial increase in friction with velocity followed by a decrease for $v > v_r$.

Testing devices capable of shearing soil samples to velocities in excess of 0.1 m/s have been developed in the past decade. Temperature in these tests rises to values that can cause transformation of minerals. As a rough guide, montmorillonite loses water molecules from interlayers at temperatures <200 °C; kaolinite loses water and transforms into a complex

amorphous structure of aluminum and silica compounds at about 550°C and calcite transforms into CaO and releases CO₂ gas at temperatures of 600°C - 850°C.

Ujiie and Tsutsumi (2010) report the results of tests performed on clayey gouges in a rotary shear apparatus capable of measuring the shearband temperature. In saturated samples, temperature increase and reduction of apparent friction under high shearing velocity (in excess of 0.1mm/s) develop in tandem. The authors attributed the loss of friction to the thermally induced dilation of water and concluded that mineral de-hydration and water vaporization were not responsible for the rapid “slip weakening” observed. For shearing rates below 0.1 mm/s it was found that the clay gouge exhibited friction strengthening; this is consistent with data reported by Tika et al (1996).

Many authors have reported in recent years the results of high velocity shearing (limited to around 1.3m/s in most cases) in a variety of soil types (Di Toro et al, 2006; Mizoguchi et al, 2007; Ferri et al, 2010; Liao et al 2011; Han and Hirose, 2012; Yang et al, 2014) tested both saturated and unsaturated. The soils tested are typically a mixture of quartz, carbonate minerals and clay minerals. Pore pressures and temperatures were, apparently, never measured. These tests as well as field observations (e.g. the presence of sheared mud in joints adjacent to the main shearing surface; the observation of molten rock injected from the sliding surface into the fault after earthquakes) indicate that the shearing strength reduction may have the following origins: grinding of minerals into nanoparticles of low friction; de-hydration of minerals and the associated increase in pore pressure; de-carbonation of calcite and the release of pressurized gas; melting of minerals, resulting in a viscous material and thermal pressurization because of the different dilation coefficients of water and minerals, in the case of saturated soils. Lack of saturation enhances the transformation of clay minerals and the release of CO₂, which is an indication of unsaturated samples attaining higher temperatures compared with saturated ones. The results can be explained by considering that saturation

allows the reduction of normal effective stress to very small values, in turn reducing heat production despite the high shearing rate. This is consistent with the results of Ujiie and Tsutsumi (2010) mentioned above.

Shearing rates in creeping slides are not expected to exceed 0.1-1 mm/s in practice. Therefore the notion of creeping can be explained by friction strengthening with increasing shearing rate, without the need to invoke the concept of viscosity. On the other hand, the extension of friction-rate relationships such as equations (6), (7) or (8) to high shearing rates is more uncertain.

The question of the transformation of a creep motion into an accelerated motion eventually leading to failure has also been approached from a different perspective. Monitoring of slope deformations suggests that the time to failure may be predicted by relating linearly the inverse of velocity with time (Saito, 1965, 1969; Voight, 1978). Helmstetter et al (2003) discuss the rationale behind these procedures. They further examine the effect of friction strain rate and state dependence by considering the dynamics of a simple sliding block in conjunction with the friction law given by equation (6), elaborated as:

$$\mu = \mu_0 + A \ln \frac{\dot{\delta}}{\dot{\delta}_0} + B \ln \frac{\xi}{\xi_0} \quad (9)$$

where μ is the friction coefficient, $\dot{\delta}$ is the sliding velocity, ξ a state variable, subscript 0 refers to reference values and A and B are model parameters. The evolution of the state variable is given by:

$$\frac{d\xi}{dt} = 1 - \frac{\xi \dot{\delta}}{D_c} \quad (10)$$

where D_c is interpreted as a critical size of asperities of the shearing surface. Variable ξ may model strengthening actions (aging, for instance) or weakening effects for accumulated shear displacements. The authors concluded that the stability of the motion is controlled by the ratio B/A : $B/A > 1$ leads to instability, where the slope velocity increases continuously. They suggest

that this was the case of Vaiont. If $0 < B/A < 1$ the motion remains stable and the velocity may increase or decrease towards a constant value. They suggest that this is the case of La Clapière (Follacci et al, 1993). The result is very attractive because it seems to provide a simple and elegant solution to a complex problem. However, this model is unable to explain the fast acceleration of the landslide once it has initiated, i.e. the transition from pre-failure velocities of the order of 4cm/day to a final velocity of 30m/sec in an estimated time of 15s. It is concluded that a different physical phenomenon should be explored if such acceleration is to be captured. The situation is sketched in Figure 2. The rate and state dependent laws of friction seem to be capable of predicting creeping behaviour and even a (slow) transition towards failure (a phenomenon also known as tertiary creep). But they are not capable of explaining very fast slide accelerations starting at a state of creeping velocities. Modelling this transition is the subject of the next section.

In the remaining of this paper the following rate dependent friction angle will be used:

$$\tan \phi = \tan \phi_0 \left(1 + A \ln \frac{v}{v_0} \right) \quad (10)$$

where v is the sliding velocity across the shear band and v_0 is a reference velocity. Interpretation of the reported results on residual friction of clayey soils indicates that the rate parameter A can be taken to vary between extremes of 10^{-5} and 10^{-2} . For a baseline friction of 12° and a reference velocity $v_0 = 0.01 \text{ mm/min}$, this range implies that a four-orders-of-magnitude increase of velocity will lead to an increase in the residual friction angle between 0.01° for $A = 10^{-5}$, which is negligible, and 5.7° for $A = 10^{-2}$, which represents an extreme, near-50% increase over the baseline value.

3. A THM STRAIN RATE DEPENDENT SLIDING BLOCK MODEL. PROBLEM FORMULATION

Problem Description

Consider a slope inclined an angle β and a planar landslide with constant thickness D (Fig. 3). The sliding mass moves as a rigid body and deformations concentrate in a shear band of thickness $2e$ parallel to the slope surface (Fig.3). The shearing zone is a large planar surface compared with the band thickness, undergoing uniform deformation. A linear profile of shearing velocity inside the band is assumed. Temperature and pore pressure build-up will be concentrated into the shear band and its vicinity. Therefore the analysis may be formulated as one-dimensional in the direction normal to the band. Water and energy transfer become one-dimensional processes along the spatial coordinate z (Fig.3). The band thickness is typically small, in the range of a few mm or cm. Shear bands in real cases are often embedded in thicker layers of claystone or clay bearing rocks, whose properties the band material may be assumed to share.

Water flow parallel to the slope, over height h_w above the sliding plane, is assumed (Fig. 3).

Dimensionless Governing Equations

The landslide motion is described using the dynamics equation. Acceleration results from the difference between gravitational driving forces and frictional resistance forces along the shear band. Frictional forces are controlled by the pore water pressure which is equal to the sum of the initial pore water pressure, present before movement initiates, and any excess pore water pressure generated during sliding due to shearing induced heating. Pore pressure and heat dissipation occur simultaneously and are governed by the corresponding mass and energy balance equations inside and outside the shear band.

The following dimensionless variables have been selected to write the governing equations:

$$\hat{z} = \frac{z}{D}, \hat{t} = \frac{t\sqrt{gD}}{D}, \hat{\theta}(\hat{z}, \hat{t}) = \frac{\theta(z, t)}{\theta_0}, \hat{u}_w(\hat{z}, \hat{t}) = \frac{u_w(z, t)}{\rho g D}, \hat{p}_w = \frac{p_w}{\rho g D}, \hat{v}(\hat{t}) = \frac{v(t)}{\sqrt{gD}} \quad (11)$$

where t is the time, θ the temperature, u_w the excess pore water pressure, p_w the initial pore water pressure and v the landslide velocity. $\rho = n\rho_w + (1 - n)\rho_s$ is the density of the saturated soil, expressed in terms of porosity, n , and the densities of water ρ_w and solid particles ρ_s . g is the gravitational acceleration and θ_0 is a reference initial temperature, assumed equal to 10°C in the calculations reported below. The governing equations and initial and boundary conditions of the problem are expressed as follows:

Dynamic equilibrium:

$$\frac{d\hat{v}(\hat{t})}{d\hat{t}} = \sin(\beta) - \left(\cos(\beta) - \frac{\hat{p}_{wh}}{\cos(\beta)} - \frac{\hat{u}_w^{\max}(\hat{t})}{\cos(\beta)} \right) \tan \phi' \quad (12)$$

Initial condition: $\hat{v}(\hat{t} = 0) = 0$

Energy balance equation:

$$\frac{\partial \hat{\theta}(\hat{z}, \hat{t})}{\partial \hat{t}} - \frac{\Gamma}{(\rho c)_m D \sqrt{gD}} \frac{\partial^2 \hat{\theta}(\hat{z}, \hat{t})}{\partial \hat{z}^2} = \hat{H}(\hat{z}, \hat{t}) \quad (13)$$

where

$$\hat{H}(\hat{z}, \hat{t}) = \frac{\rho g D^2}{(\rho c)_m \theta_0 2e} \left(\cos^2(\beta) - \hat{p}_{wh} - \hat{u}_w^{\max}(\hat{t}) \right) \tan \phi' \hat{v}(\hat{t}) \text{ for } \hat{z} \in [0, e/D] \quad (14a)$$

$$\hat{H}(\hat{z}, \hat{t}) = 0 \text{ for } \hat{z} \in (e/D, L_0] \quad (14b)$$

427 Initial and boundary conditions are given by:

$$\hat{\theta}(\hat{z}, \hat{t} = 0) = 1 \quad (15a)$$

$$\left. \frac{\partial \hat{\theta}}{\partial \hat{z}} \right|_{(\hat{z}=0, \hat{t})} = 0 \quad (15b)$$

$$\hat{\theta}(\hat{z} = e^-/D, \hat{t}) = \hat{\theta}(\hat{z} = e^+/D, \hat{t}) \quad (15c)$$

$$\left. \frac{\partial \hat{\theta}}{\partial \hat{z}} \right|_{(\hat{z}=e^-/D, \hat{t})} = \left. \frac{\partial \hat{\theta}}{\partial \hat{z}} \right|_{(\hat{z}=e^+/D, \hat{t})} \quad (15d)$$

$$\hat{\theta}(\hat{z} = L_0/D, \hat{t}) = 1 \quad (15e)$$

428 The specific heat of the saturated porous medium is calculated using a weighted average:

429 $(\rho c)_m = n\rho_w c_w + (1-n)\rho_s c_s$ where c_w and c_s is the specific heat of water and solid particles

430 respectively. Fourier's thermal conductivity is also calculated as a weighted average

431 $\Gamma = n\Gamma_w + (1-n)\Gamma_s$, where Γ_w and Γ_s the conductivity of water and solid respectively. In line

432 with (Alonso and Pinyol, 2010b), advective heat transfer is neglected.

433 The dimensionless source term $\hat{H}(\hat{t})$ in equation 14a quantifies the heat input into the shear

434 band in terms of the work consumed per unit volume of the material by its shear strength.

435 Outside the band, where the mass moves as a rigid body, no heat is generated (eq. 14b).

436 Regarding initial and boundary conditions, the first condition (eq. 15a) imposes that initially

437 the shear band and surrounding zone are at the reference temperature. No heat is transferred

438 across the center of the shear band and therefore no heat flow is imposed at $\hat{z} = 0$ (eq. 15b).

439 Continuity of temperature and temperature flow should be satisfied on both sides of the shear

440 band (eq. 15c and 15d). The temperature boundary condition at $\hat{z} = L_0/D$ (eq. 15e) stipulates

441 that temperature remains unaffected beyond a pre-determined distance L_0 from the axis of

442 the shear band. If taken further away from the shear band than heat flow will reach during the

443 time frame modelled, L_0 does not affect the results.

444 *Mass balance equation:*

$$-\frac{\beta_{soil}\theta_0}{m_{soil}\rho g D} \frac{d\hat{\theta}(\hat{z}, \hat{t})}{d\hat{t}} + \frac{d\hat{u}_w(\hat{z}, \hat{t})}{d\hat{t}} - \frac{k}{\gamma_w m_{soil} D \sqrt{g D}} \frac{d^2 \hat{u}_w(\hat{z}, \hat{t})}{d\hat{z}^2} = 0 \quad (16)$$

Initial and boundary conditions are given by:

$$\hat{u}_w(\hat{z}, \hat{t} = 0) = 0 \quad (17a)$$

$$\left. \frac{\partial \hat{u}_w}{\partial \hat{z}} \right|_{(\hat{z}=0, \hat{t})} = 0 \quad (17b)$$

$$\hat{u}_w(\hat{z} = e^-/D, \hat{t}) = \hat{u}_w(\hat{z} = e^+/D, \hat{t}) \quad (17c)$$

$$\left. \frac{\partial \hat{u}_w}{\partial \hat{z}} \right|_{(\hat{z}=e^-/D, \hat{t})} = \left. \frac{\partial \hat{u}_w}{\partial \hat{z}} \right|_{(\hat{z}=e^+/D, \hat{t})} \quad (17d)$$

$$\hat{u}_w(\hat{z} = L_{u_w}/D, \hat{t}) = 0 \quad (17e)$$

In the mass balance equation (16), β_{soil} is a weighted average thermal expansion coefficient for saturated soils calculated as: $\beta_{soil} = n\beta_w + (1-n)\beta_s$, where β_w and β_s are the thermal expansion coefficients of water and solid particles respectively. These depend on temperature, e.g. Wagner and Kruse (1998); however as a first approximation they are assumed here to be constant. m_{soil} is the compressibility of saturated soil, calculated as $m_{soil} = m_v + n\alpha_w$ where m_v and α_w are the coefficients of compressibility of the soil skeleton and the water respectively. k is the saturated permeability of the soil.

Initial and boundary conditions are similar to those imposed for heat transfer. The initial value of excess pore pressure is zero (eq. 17a). No water flow crosses the center of the shear band (eq. 17b). Equations 17c and 17d ensure continuity of excess pore pressure and flow rate on both sides of the band. The excess pore pressure is assumed zero beyond a pre-determined distance L_{u_w} . If this distance is far enough for excess pore pressure to not develop there, it does not affect the results; otherwise it stipulates the existence of a drainage boundary.

461 L_θ and/or L_{u_w} may be interpreted as the thickness of the claystone layer containing the shear
 462 band: pore pressure and temperature should be maintained constant at its outer boundary.
 463 Alternatively, if the layer is in contact with a pervious rock mass, it is reasonable for these
 464 conditions to apply at the interface.

465 Four dimensionless coefficients are naturally defined in the balance and equilibrium equations.
 466 They are associated with the physical phenomena describing the problem:

467 $\Theta = \frac{\Gamma}{(\rho c)_m D \sqrt{gD}}$, coefficient associated with heat flow in the heat balance equation;

468 $\Psi = \frac{\rho g D^2}{(\rho c)_m \theta_0 2e} = \frac{\rho D (\sqrt{gD})^2}{(\rho c)_m \theta_0 2e}$, coefficient of the source term in the heat balance equation;

469 $\Pi = \frac{\beta_{soil}}{m_{soil}} \frac{\theta_0}{\rho g D}$, coefficient associated with thermal expansion of porous rock in the mass
 470 balance equation of water;

471 $\Sigma = \frac{k}{\gamma_w m_{soil} D \sqrt{gD}}$, coefficient associated with stress induced volumetric deformation in the
 472 mass balance equation of water;

473 Θ is a measure of the thermal dissipation and combines thermal conduction, heat storage, a
 474 reference dimension and a reference velocity; Ψ can be interpreted as the ratio between the
 475 kinetic energy of the moving mass and the initial heat stored in the shear band, Π is a ratio of
 476 the thermal expansion of the saturated porous medium with respect to its mechanical
 477 compressibility and Σ is a dimensionless consolidation coefficient that combines permeability,
 478 confined compressibility and the sliding depth.

479 For a given planar landslide of thickness D , the range of variation of the non-dimensional
 480 parameters is limited. Properties such as density, Fourier's coefficient, thermal expansion

coefficient and the specific heat for common soils and rocks exhibit small variation especially if compared with other parameters such as permeability. Typical values for these material properties are indicated in **Error! No s'ha trobat l'origen de la referència..** On the contrary, permeability, k , soil compressibility, m_v , and thickness of the shear band, e , may change more significantly. Typical ranges of values of these variables are collected in **Error! No s'ha trobat l'origen de la referència..** As a consequence, the dimensionless parameters that may vary significantly depending on the particular case analyzed are: Ψ which is inversely proportional to the band thickness; Π which is inversely proportional to the band compressibility coefficient; and Σ which depends on the ratio of permeability and soil compressibility.

Using **Error! No s'ha trobat l'origen de la referència.** and **Error! No s'ha trobat l'origen de la referència.,** ranges for the values of the non-dimensional parameters can be established. Θ is essentially constant and equal to $1.27 \cdot 10^{-9}$. Ψ depends on the shear band thickness and may vary between 30 and 1800. Π may vary by two orders of magnitude, roughly between 0.1 and 10, due to the variability of the shear band compressibility coefficient. Finally, Σ depends on the ratio between permeability and compressibility so its range of variation is much wider, between 10^{-3} and 10^7 . Shear bands are often located in highly overconsolidated layers having a wide range of clay mineral content. Their compressibility will be closer to the medium and low values within the range indicated in Table 2, reducing the variability mainly of Π but also of Σ in practice. Permeability remains as the main source of variability.

Zero thickness shear band

Idealizing the sliding surface as a zero thickness shearing band may be sufficiently close to actual conditions, especially in highly plastic materials where sliding surfaces are often described as smooth and polished planes. In terms of problem formulation and its numerical solution, the hypothesis of zero band thickness presents some advantages: a) the geometry is simplified and one of the parameters entering the dimensionless formulation, the band

506 thickness, $2e$, disappears and b) the discretization of the domain is simplified because there is
 507 no need to discretize the very thin band.

508 The rate of frictional work expended along the contact plane, per unit of surface, is now
 509 calculated as the product between shear strength and velocity. This mechanical work
 510 dissipates into heat that enters the rock mass. With reference to Figure 1, when $2e=0$, the
 511 coordinate $z=0$ indicates the position of the contact plane.

512 The system of equations governing the problem is similar to the system for finite band
 513 thickness. The difference lies in the heat balance equation, which becomes:

$$514 \quad \frac{\partial \hat{\theta}(\hat{z}, \hat{t})}{\partial \hat{t}} - \frac{\Gamma}{(\rho c)_m D \sqrt{gD}} \frac{\partial^2 \hat{\theta}(\hat{z}, \hat{t})}{\partial \hat{z}^2} = 0$$

515 with the following initial and boundary conditions:

$$\left\{ \begin{array}{l} \hat{\theta}(\hat{z}, \hat{t} = 0) = 1 \\ \left. \frac{\partial \hat{\theta}}{\partial \hat{z}} \right|_{(\hat{z}, \hat{t} = 0)} = 0 \\ \frac{1}{2} H(\hat{t}) = -\frac{\Gamma \theta_0}{D} \left. \frac{\partial \hat{\theta}}{\partial \hat{z}} \right|_{(\hat{z}=0, \hat{t})} \Rightarrow \frac{\partial \hat{\theta}}{\partial \hat{z}}(\hat{z}=0, \hat{t}) = -\frac{\rho g D^2 \sqrt{gD}}{2 \Gamma \theta_0} (\cos^2(\beta) - \hat{p}_{wh} - \hat{u}_w^{\max}(\hat{t})) \tan \varphi' \hat{v}(\hat{t}) \\ \hat{\theta}(\hat{z} = L_0/D, \hat{t}) = 1 \end{array} \right.$$

516 Note that no heat is generated in the sliding mass, but heat flow is imposed through a
 517 boundary flux at $z = 0$. Because of symmetry only half of the heat generated is considered.
 518 The dimensionless parameters describing this problem remain identical to the case of finite
 519 band thickness except for the coefficient associated with the source term in the heat balance

520 equation, which becomes $\Psi = \frac{\rho g D^2 \sqrt{gD}}{2 \Gamma \theta_0}$. The heat generated at the boundary is now

521 calculated as:

$$522 \quad H(\hat{t}) = \rho g D \sqrt{gD} (\cos^2(\beta) - \hat{p}_{wh} - \hat{u}_w^{\max}(\hat{t})) \tan \varphi' \hat{v}(\hat{t})$$

Rate effects

Slow maintained slide displacement rates (creeping) will be explained by considering strain rate effects on strength, following the discussion in Section 2. The analysis concentrates on active slides irrespective of their initial creeping rate; therefore frictional strength is characterized by a residual friction angle. The effective residual friction strength is defined as the sum of the strength available at velocities lower than a given value of reference (v_{ref}) and an additional term that depends on sliding velocity:

$$\tan \phi = \tan \phi_0 + f_v \quad (18)$$

Recalling the logarithmic law of Equation (10):

$$f_v = A \ln \left(\frac{v}{v_{ref}} \right) = A \ln \left(\hat{v} \frac{\sqrt{gD}}{v_{ref}} \right) \text{ for } v > v_{ref}; \hat{v} > \hat{v}_{ref} = v_{ref} / \sqrt{gD} \quad (19a)$$

$$f_v = 0 \text{ for } v \leq v_{ref}; \hat{v} \leq \hat{v}_{ref} = v_{ref} / \sqrt{gD} \quad (19b)$$

The slope angle β and the parameters defining the friction law ($\phi_0, A, v_{ref} / \sqrt{gD}$) complete the set of dimensionless parameters governing the problem.

4. NUMERICAL INTEGRATION

Equations (12), (13) and (16) form a system of coupled, non-linear partial differential equations that needs to be solved. The use of centred-space forward-time (explicit) finite differences has been the norm in past work (e.g. Vardoulakis, 2002, Alonso et al, 2010, Cecinato et. al. 2011, 2012). However explicit integration requires very small timesteps, typically of the order of 10^{-5} sec, for numerical stability. Although acceptable when modelling the catastrophic phase of a slide over its final few tens of seconds, this onerous condition makes modelling of the long-term creep behaviour that may precede failure impractical. The alternative we explore here is

the use of an unconditionally stable implicit scheme that will allow much larger timesteps. The ability to adapt the timestep during the calculation is also essential, so that long periods of creep where the solution changes by very little can be modelled efficiently. We use here the trapezoidal rule, which is of order $O(\Delta t^2)$. In principle it is also possible to use a higher order, more accurate multistep method; however adaptive time stepping will only be possible at the cost of significantly increased complexity, e.g. Skeel (1986).

The resulting system of equations must be solved iteratively to calculate at the end of each timestep the slide velocity as well as the temperature and excess pore pressure everywhere in the domain. Solution using a full Newton-Raphson method is however problematic, as the logarithmic friction law renders the dynamics equation highly nonlinear. Convergence requires timesteps typically smaller than 10^{-3} sec, defeating the purpose of the implicit scheme.

The strategy adopted was to use a staggered solution scheme: the diffusion equations were solved by Newton-Raphson using the current approximation of the velocity; the dynamics equation was subsequently solved by Pegasus (Dowell and Jarratt, 1972), a method of the *regula falsi* family that is known to be fast and efficient in finding roots of nonlinear equations. Iterations continued till the residuals of all equations were below tolerance. The timestep was increased or decreased based on the rate, or lack of, convergence. The maximum timestep used was 24h. This approach allowed timesteps of the order of 10^5 sec for the most part of an analysis, making the efficient modelling of years of creeping motion possible.

An additional challenge was the location of the far boundary L_θ and L_{u_w} . Previous work considered that ambient values of temperature and excess pore pressure are applicable at a distance equal to a small multiple of the shearband thickness. Although this is valid for rapid heat generation during the final collapse, modelling the creep phase should require a larger domain, as heat and pore water will potentially have the time to travel longer distances. In practice, the presence of a geological feature, such as an interface with pervious rock, may

provide a natural boundary within the sliding mass, as discussed previously. For the general case, however, the following numerical approach was adopted: The far boundary was initially considered at a distance 10 times the shearband thickness. If, during the calculation, the value of either the temperature or the excess pore pressure next to the boundary was found to differ by more than 10^{-10} °C or kPa respectively from the corresponding boundary condition, the domain was expanded by 100 times the shearband thickness before the calculation continued. Therefore the problem progressively became computationally more expensive; typical calculations started with about 300 unknowns but eventually involved in excess of 300,000. Nevertheless, due to the direct, sparse multi-frontal solver (HSL, 2015) used, all models could run on a desktop computer within hours.

5. ANALYSIS

5.1 An illustrative example

The case presented below illustrates the capability of the model to integrate a history of creeping motions and the eventual triggering of a rapid catastrophic motion. Basic parameters are given in Table 1. The sliding mass is defined by $D=25\text{m}$, $h_{w0}=11.05\text{m}$, $\beta=9.8^\circ$, $\phi_0=12^\circ$, $2e=2.5\text{mm}$, $m_v=1.5\cdot 10^{-9}\text{Pa}^{-1}$ and $k=10^{-9}\text{m/s}$. The strain rate effect is characterized by $A=0.014$ and $v_{ref}=10^{-5}\text{m/s}$. An initial temperature $\theta_0=10^\circ\text{C}$ is assumed. The shear band is centered in a clayey layer 2m thick, bounded by pervious rock that is able to maintain the initial pore pressure and temperature.

The slope is initially in strict equilibrium with zero velocity. It is destabilized by increasing the water level to $h_w=12\text{m}$. The subsequent history of water level changes is given in Figure 4a. There is a transient reduction of h_w from 13m to 12.5m and a final increase to 14m. During this final stage the slide suddenly accelerates. Figure 4 provides the calculated evolution of temperature (Fig 4b,c), excess pore water pressure (Fig 4d), velocity (Fig 4e), displacement (Fig

4f) and friction angle (Fig 4g). Temperature and excess pore pressure increase in the shear band as soon as the slide becomes unstable. The sliding velocity slowly increases; however strain rate effects contribute to maintaining dynamic equilibrium. The subsequent reduction of water level reduces the sliding velocity and the shear band excess pore pressure and temperature. Similar changes occur to the friction angle, Fig. 4g, because of strain rate effects. It can be considered that the slope maintains a safety factor $SF=1$ during the entire creeping period.

When a further increase of water level to $h_w=14$ m is imposed at $t=15000$ s, temperature and pore water pressure increase rapidly and strain rate effects are no longer capable of maintaining a state of slow creeping velocity. The slide accelerates because the pore pressure reaches values close to the normal effective stress on the sliding surface, rapidly reducing the frictional resistance there.

Thermal pressurization affects the behaviour of the slide throughout. However there is a qualitative change when friction increase, induced by rate effects, is no longer capable of counteracting the strength reduction caused by pore pressure build-up in the shear band. At that time the creeping stage ends and the slope enters a high acceleration and high velocity stage promoted by a thermally-dominated “feedback loop”: temperature increase-pore pressure increase-strength reduction-velocity increase-temperature increase. From that point onwards strain rate effects no longer play a significant role in the evolution of the slide.

The remainder of this chapter explores the relationship between these two regimes of slope behaviour. They are intimately linked and particular cases characterized by a sliding geometry and material properties would require a specific analysis. However, the simple planar slide offers insights on the creep-fast sliding coupling. The dimensionless formulation also enhances the generality of the conclusions reached.

5.2 Velocity regimes in a planar slide

Results will be presented for a fixed displacement equal to twice the height of the sliding mass. This is an arbitrary limit, however analyses of longer runouts are unlikely to be representative due to topographical constraints and/or the resulting fragmentation of the sliding mass.

Rate independent friction

We consider again the previous example, this time with rate independent frictional strength. No creeping stage is now possible. This case serves as a reference for the rate dependent case.

The landslide is destabilized by imposing an increment of the dimensionless pore water pressure (\hat{p}_{wh}) equal to 0.0016, equivalent to increase the water level in 10 cm. It involves a reduction of the SF from SF=1 to SF= 0.9978.

The effect of the shear band thickness is first examined. This parameter only affects Ψ , the dimensionless coefficient of the source term in the energy balance equation. **Error! No s'ha trobat l'origen de la referència.** shows the maximum dimensionless velocity for different values of Ψ and Σ and a constant $\Pi=1$ value. Since Π remains constant for all the cases analyzed, variation of Σ implies essentially variation of the permeability. The two plots in Figure 5 present the same information in two different ways. The nearly horizontal lines in Figure 5a indicate that the effect of parameter Ψ on the velocity reached is small. This is also apparent from Figure 5b, where the calculated velocities for different band thicknesses essentially overlap. Parameter Ψ directly affects the heat generated in the shear band. The higher its value, i.e. the smaller the band thickness, the larger the quantity of heat generated. However, smaller band thickness also means easier dissipation of the heat and excess pore pressure generated within the band towards the surrounding rock mass. Conversely, a thicker band will produce less heat and excess pore pressure, which will take longer to dissipate. The maximum velocity reached is similar to the calculated velocity for a case in which no thermal effects are included (also plotted in Figure 5). For values of Σ lower than 10^{-7} , no effect of Ψ and Σ is observed and the maximum velocity is the same in all cases. The case of zero

thickness shear band is also included in the plot. No significant differences are observed when compared with finite thickness bands.

Parameter Σ , which controls the dissipation of excess pore pressure is key in determining the response of the slide. Values of Σ above 10^{-3} result in almost no heat-induced effect. Thermally induced excess pore pressure can dissipate quickly in these cases and frictional strength remains slightly smaller than the initial value. The ratio between the shear strength remaining after the maximum runout has been reached and the initial shear strength is plotted in Figure 6. The shear strength essentially vanishes for values of Σ below 10^{-7} , while for values above 10^{-3} it maintains its initial value.

It is concluded that the effect of the dimensionless parameter Ψ , which essentially varies with the thickness of the shear band, is quite limited.

Parameter Σ combines permeability and band compressibility. However the effect of compressibility cannot be accounted for exclusively by Σ because parameter Π is inversely proportional to compressibility: it controls the development of pore pressures for a given heat input, which is an effect independent from pore pressure dissipation, which is governed by Σ . An additional set of calculations were performed to show the combined effect of permeability and compressibility for constant $\Psi=345$, which corresponds to a shear band thickness $2e=5\text{mm}$. The selected values for parameter Π imply a wide, about two orders of magnitude, range of variability of the soil compressibility. The values of **Error! No s'ha trobat l'origen de la referència.** are used for the remaining parameters. The results are plotted in Figure 7.

Figure 7 shows, as well as Figure 5b for $\Pi=1$, that there is a range of dimensionless consolidation coefficients Σ that separates a high velocity regime (for low Σ values) from a low velocity-, no-heat effect regime (for high Σ values). The observed trends in the variation of velocity with parameter Σ are similar for all Π values. Reducing Π , i.e. increasing band

stiffness, decreases the threshold Σ range separating high and low velocity regimes. In other words, given a value of Σ , lower values of Π , which control the heat induced pore water pressure, result in lower sliding velocity.

Rate dependent friction

Consider now the effect of rate dependent friction. Material properties given in **Error! No s'ha trobat l'origen de la referència.** have also been adopted in the cases presented in this section.

A base case for discussion is defined by using the frictional law of equations (18) and (19), for $A = 4.7 \cdot 10^{-3}$ and a reference velocity $\hat{v}_{ref} = 6.4 \cdot 10^{-7}$. The effect of these parameters on the gain in strength with velocity is plotted in Figure 8. This rate dependent law is within the range of experimental results measured at low to moderate shearing velocities (say, below 0.1m/s). Above this velocity thermal pressurization and other phenomena discussed before contribute to the measured rate dependence. However, the plot is extended to high velocities to show the maximum contribution which may be expected from rate effects at high shearing speeds. The parameters chosen imply an increment of 7%, close to one degree, for high sliding velocity (30 m/s). The rate dependent strength increases quickly at low velocity and then levels off. Results for a different value of parameter A , used in the discussion below, are also plotted in Figure 8.

The slide is destabilised by imposing a 1m increment of the water level. This reduces the SF from 1 to 0.98. Limit equilibrium for this water level is reached for a friction angle equal to 12.32°, 2.7% higher than the static friction angle associated with velocities lower than the reference creeping velocity.

The base case was analyzed for different values of the Π and Σ parameters. The band thickness-controlled parameter Ψ is considered constant and equal to 345, as its effect has been shown to be insignificant. The results in terms of maximum dimensionless velocity are plotted in **Error! No s'ha trobat l'origen de la referència.** The case where thermal effects are

692 ignored is added to the figure; in that case dynamic equilibrium is reached for a dimensionless
693 velocity of $4.6 \cdot 10^{-5}$ equivalent to $7.2 \cdot 10^{-4}$ m/s, a reference velocity that depends only on the
694 safety factor attained immediately after the initial instability (Equation 1).

695 When thermal effects are considered, the response observed is similar for the different values
696 of parameter Π selected. A narrow range of Σ values separates fast and slow sliding regimes. If
697 compared with the no-rate-effect case (Figure 5b) the calculated sliding velocities are now
698 higher. This is a consequence of the stronger initial trigger imposed to initiate the slide run-out
699 (Δh_w is now 1 m against $\Delta h_w=0.1$ m in the previous case).

700 The effect of parameter Π is similar to the effect observed in cases where rate effects were not
701 included. However, the threshold range of Σ values separating the slow and fast sliding
702 regimes decreases when a rate effect on friction is included. This becomes apparent by
703 comparing Figures 9 and 7. In other words, rate effects extend the range of “safe” cases, i.e. of
704 slopes that will not accelerate due to thermal pressurization effects. It appears also that the
705 threshold Σ range is narrower in Figure 9 (rate effects included), compared with Figure 6 (no
706 rate effects)

707 Figure 10 presents the maximum temperature in the centre of the shear band for the cases
708 plotted in Figure 9. If the landslide reaches a high velocity quickly, as is the case for Σ lower
709 than 10^{-9} , the shearband temperature remains relatively low. The temperature also remains
710 low, almost constant, in those cases where the velocity remains low, in a creeping mode, when
711 Σ is higher than 10^{-5} . Maximum values of temperature are reached in intermediate cases,
712 where the excess pore pressure rises relatively slowly and the velocity, although it may
713 eventually become very high, increases at a slower pace. The acceleration of the slide is in fact
714 a critical factor controlling the development of temperature.

Increasing the rate dependent component of friction has been shown to have a significant effect in reducing the threshold permeability (through Σ) that leads to a fast sliding regime. This is further shown in Figure 11 where the maximum velocity is plotted in terms of Σ for three A values (see Figure 8 for their effect on friction increase). In all cases $\Psi=345$ and $\Pi=1.0$. As expected, the creep velocity when thermal effects are ignored (or Σ has a high value) decreases for increasing A . It is noted that for relatively low values of Σ , when excess pore pressure dissipation is slow, the maximum velocity attained is similar in all cases irrespective of the considered rate dependence of friction. When thermal effects develop they dominate the sliding behaviour. However, the development of heat induced pore pressure build-up depends on the creeping history. This relevant issue is discussed in more detail in the next section.

5.3 Evolution of motion. Creep-thermo-mechanical interactions

Attention is now focused on the effect of the rate of friction increase on the blow-up time, i.e. the time after initiation at which the slope enters a final catastrophic phase. Calculations are performed for a planar slide with $D=240\text{m}$, base friction angle of 12° and slope angle of 9.8° . An unbounded domain is assumed and a maximum runout of $2D=480\text{m}$ is considered.

To investigate the effect of the rate parameter A , a set of analyses is run where the slide is triggered by a minuscule increase of h_w by 0.025% above the strict equilibrium value, resulting to a change in the safety factor $\delta SF = 6 \cdot 10^{-5}$. In the case of rate-independent friction (i.e. $A=0$) catastrophic failure occurs immediately; the slide is classified as “rapid” (IUGS, 1995) 2sec after initiation and, within a few more seconds, frictional heating leads to high temperature and excess pore pressure almost equal to the overburden stress.

The introduction of even a very mild rate effect, e.g. $A=10^{-5}$ corresponding to a negligible increase in friction angle by 0.01° for a 10^4 -fold increase in velocity, suppresses catastrophic acceleration and leads to creep at constant velocity. The magnitude of the velocity attained is

governed by the rate parameter A ; for $A=10^{-5}$ it is 22cm/day and drops to 1.5cm/day for $A=10^{-3}$, classifying both slides as “slow” (IUGS, 1995). The temperature increases by 10^0C or less and the excess pore pressure generated is negligible.

To further explore the effect of the rate parameter a second set of analyses is run, where the slide is triggered by a more substantial increase of the groundwater level, resulting to $SF = 0.99$. Figure 12 summarises the results.

In the case of rate-independent friction $A=0$ but also for any $A \leq 10^{-4}$, high temperature and excess pore pressure occur almost immediately and lead to catastrophic failure. For $A = 5 \cdot 10^{-4}$ a transitional behaviour is observed, where the slide first creeps with near-constant velocity for the first 15sec before accelerating to catastrophic failure, while for $A > 5 \cdot 10^{-4}$ the slide creeps with “moderate” (IUGS, 1995) constant velocity. Therefore a threshold value A_{crit} of the rate parameter must exist, that determines whether catastrophic acceleration will occur; in this case $6 \cdot 10^{-4} > A_{crit} > 5 \cdot 10^{-4}$. This is further explored using models for a range of values in this interval. Figure 12 shows that even small changes, of the order of 2%, of the rate parameter can have order-of magnitude impact on the duration of the creep phase, although the predicted creep velocity is less sensitive. For comparison, Figure 12 also presents the baseline velocity, i.e. the constant creep velocity each slide would attain if thermal effects were ignored: it is evident that, even where thermal effects do not eventually lead to catastrophic collapse, they still increase slide velocity by at least one order of magnitude.

It is also interesting to note that it is the *rate* of temperature increase that controls excess pore pressure generation, and thus the associated loss of strength that leads to catastrophic failure.

If the time scale of heat generation is large enough the induced excess pore pressures may dissipate before thermal pressurization occurs, irrespectively of how high the temperature attained is. For a given rate of heat production, the rate of excess pore pressure generation

will depend on soil permeability. It is thus expected that permeability, therefore the value of parameter Σ , will influence both the creep velocity and the timing of a possible catastrophic failure.

To further investigate the impact of parameter Σ , a set of analyses is run with the same trigger, rate parameter $A = 5.75 \cdot 10^{-4}$ and Σ varying between $\Sigma = 6 \cdot 10^{-8}$ and $\Sigma = 6 \cdot 10^{-13}$. The results are summarised in Figure 13.

Lower values of Σ lead to faster development of excess pore pressure and earlier onset of a catastrophic phase. As Σ increases so does the duration of the pre-failure creep phase; this relationship is highly nonlinear and, as was the case for the rate parameter, a threshold value Σ_{crit} exists above which a catastrophic phase is never reached. In this case $\Sigma_{crit} \cong 6 \cdot 10^{-11}$; note how, close to that value, increasing the value of Σ by just 2% results to an increase in the duration of the creep phase by a factor of almost three. Figure 13 further shows that, for the parameters used here, the velocity of the slide during the creep phase is essentially unaffected by the permeability; the baseline velocity is again plotted for comparison.

Finally, it was found that the maximum temperature that develops in the shearband increases with permeability, regardless of whether a catastrophic phase is eventually reached. This is a direct consequence of the strong influence of permeability on the rate of excess pore pressure generation: the lower the permeability, the faster excess pore pressure rises and effective stress drops, therefore reducing the rate of energy dissipation into heat. Conversely, the higher the permeability the higher the rate of heat production necessary to achieve pressurisation; in fact, models that do not predict a final catastrophic phase consistently attain higher shearband temperature than models that do.

5.4. Compound geometry. Creep-thermo-mechanical interactions

Previous sections analyze the landslide evolution including two phenomena that work in opposite directions: the heat-driven acceleration of the landslide and the stabilizing effect of friction rate effects. In a planar geometry the slide remains always in motion. The creeping velocity, away from blowup, depends on the magnitude of the destabilization factors, the intensity of rate effects and additional properties of the shear band, notably its permeability. But other sliding geometries are common. A good example is Vaiont, which may be conceptualized as an evolutionary two-block mechanism (Pinyol and Alonso, 2010b). The sliding motion results in a reduction of the mass of the upper unstable wedge and a parallel increase of the mass of the stabilizing lower wedge. This is a self-stabilizing mechanism unlike the single block case. The effect of this major change in the kinematic description of the landslide is explored here.

Consider the two-block mechanism given in Figure 14. The interaction between the two sliding blocks is given by the axial force on the hinged “rod” connecting the blocks. Changes in water pressure act only on the lower block. In the cases solved both wedges have the same thickness but different initial length. Equations of dynamic equilibrium were written for the two interacting blocks. Balance equations (water flow, energy) were formulated for each of the two blocks, following the previous discussion. A zero-thickness band was adopted in the two blocks. The following example was analyzed: Block thickness, 25 m; the upper and lower blocks have lengths of 60 m and 120 m and slide on a plane inclined 37° and 0° respectively. The remaining physical constants are identical to the planar slides analyzed previously. The stiffness coefficient, $m_v=1.5 \cdot 10^{-9} \text{ Pa}^{-1}$. It leads to the following constant dimensionless parameters for the case analyzed: $\Theta=1.27 \cdot 10^{-9}$, $\Psi=2.72 \cdot 10^7$ and $\Pi=1$. The landslide is destabilised by increasing the water pressure on the sliding surface of the lower block (\hat{p}_{wh} increases by 0.04 which is equivalent to increasing the water level by 1m).

The calculated evolution of velocity and run-out if no friction rate effects are considered is given in Figure 15. The effect of Σ , which depends on permeability in the vicinity of the shear band, is highlighted. Low values, below $1.63 \cdot 10^{-7}$ m/s, lead to a slide blowup. This is also the case of a planar slide. However, the main difference of the compound mechanism is that the slide is able to stop, for high band permeability, as a consequence of the weight transfer between upper and lower blocks. Note also that the thermal effect results in some increase of the run-out and velocity if compared with the no thermal pressurization analysis.

Friction rate effects, for the same case, are shown in Figure 16 ($A = 4.7 \cdot 10^{-3}$). If conditions lead to thermal blowup, rate effects and slide geometry become largely irrelevant in controlling the sliding velocity and run-out. However, if blowup conditions are not attained, rate effects play a significant role: they reduce the Σ threshold between fast and creeping regimes of the slope and run-out and velocity during the transient motion of the slide after initial instability is significantly reduced. This becomes apparent by comparing Figures 15 and 16.

6. CONCLUSIONS

Known cases of rapid landslides can only be explained by negligible shear strength acting on the main failure surface. Among the proposed mechanisms of strength reduction, thermal pressurization of pore water in saturated shear bands may explain a fast accelerated motion as well as a transition from creeping to fast regimes in cases of landslide reactivation. Measured strain rate effects on friction, at low to moderate shearing rates, suggest a linear increase in friction with the logarithm of shearing velocity, a relationship supported by basic friction concepts.

A dynamic modelling of landslide motion, which integrates strain rate effects and thermal interactions in a deforming shear band, provides considerable insights into the evolution of the

836 sliding velocity and its eventual blowup, when thermal pressurization dominates the slide
837 motion. A model of this kind, initially developed for planar landslides, has been described by a
838 set of dimensionless balance and equilibrium equations. Four dimensionless constitutive
839 coefficients which control the slide movement were derived. It was found that the shear band
840 permeability and, to a lesser extent, its compressibility, dominate the entire phenomenon.
841 The set of equations require a numerical solution. A novel staggered implicit integration
842 procedure has been proposed. It is capable of handling, within reasonable computational
843 times, a long creeping history and a sudden final blowup.

844 Transition from a slow creeping-like motion to fast acceleration is quite sharp in terms of a
845 dimensionless “consolidation” coefficient which relates band permeability and stiffness as well
846 as the height of the sliding mass. This threshold is significantly affected by friction rate effects
847 in the sense that increasing rate effects reduces the range of shear band properties leading to
848 blowup phenomena. Given a creeping motion of a specific landslide, time to blowup is very
849 sensitive to strain rate effects. Beyond a certain value of the rate parameter, blowup does not
850 seem to be possible within realistic runout distances.

851 The paper also examines the behaviour of compound slide geometries, in particular those
852 consisting of a self-stabilizing mechanism. If blowup conditions develop the slide accelerates as
853 much as a planar slide. However, the creeping stage is also controlled, in addition to aspects
854 valid for planar slides, by geometrical details and the evolutionary changes in slide geometry.

855 Finally, although the intention of the paper was to provide a theoretical investigation of the
856 problem rather than an analysis of a specific landslide, the models presented can be readily
857 used on appropriate real cases, at least as a back-analysis tool. Predictive modelling is also
858 possible in principle; however a pertinent issue in this respect is the calibration of material
859 parameters, especially the rate parameter, which may have significant impact on predictions.

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1068

FIGURES

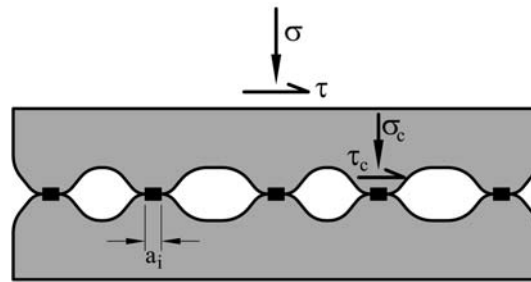


Figure 1. Sketch to illustrate the concept of friction.

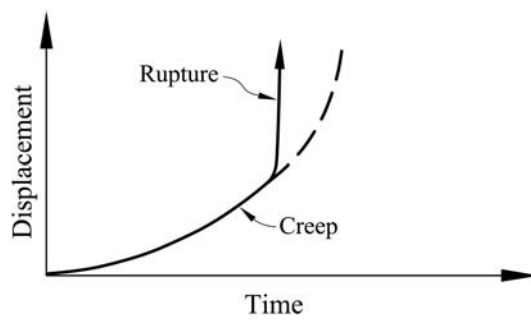


Figure 2. Creep and rupture: two interacting mechanisms in action

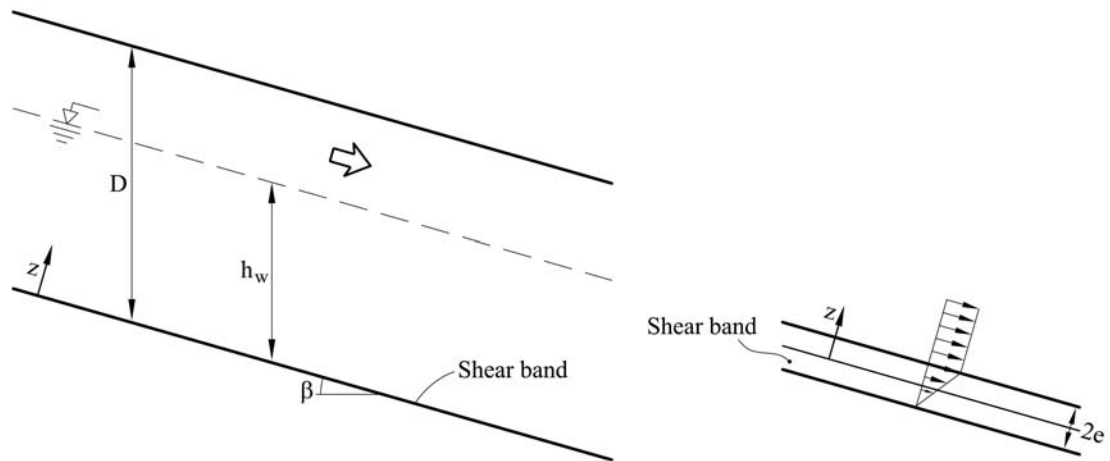
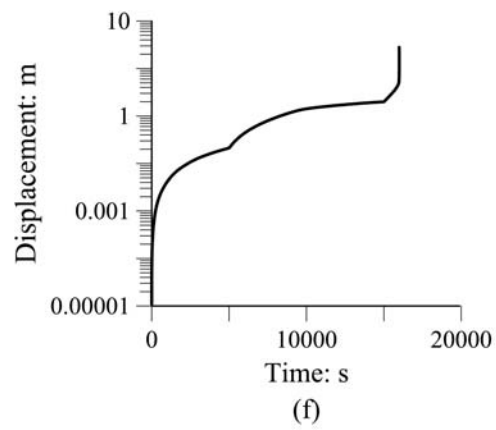
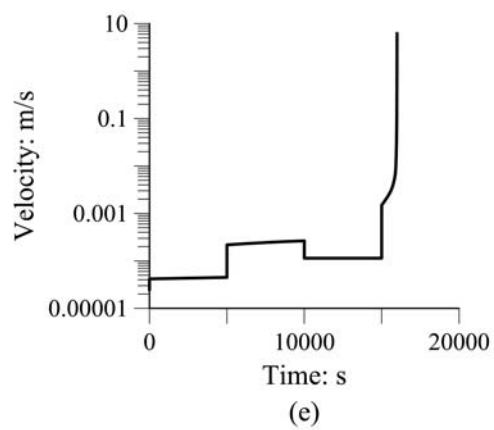
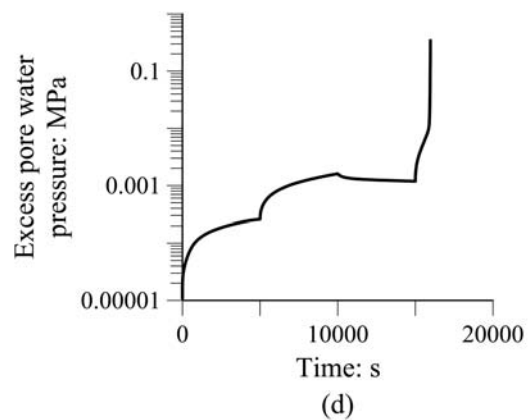
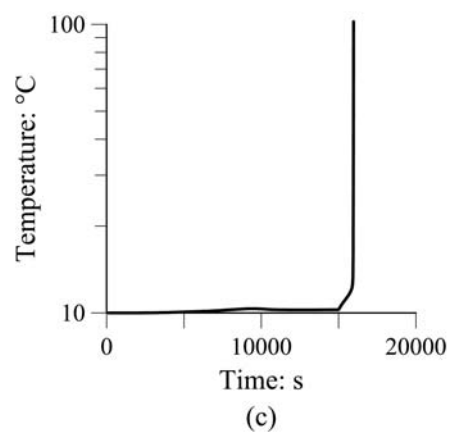
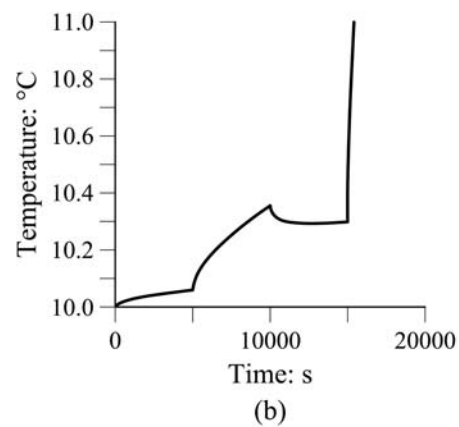
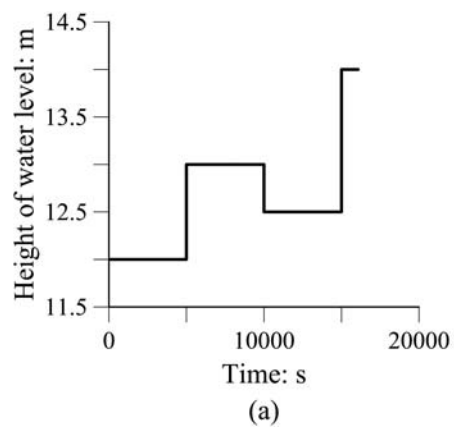
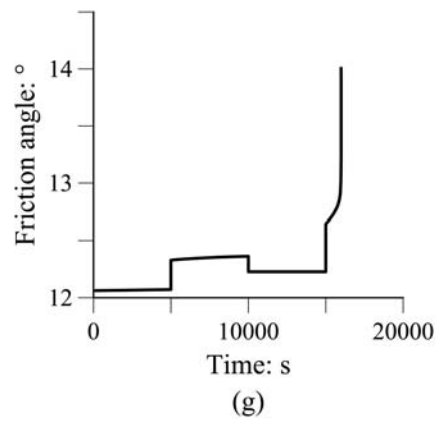


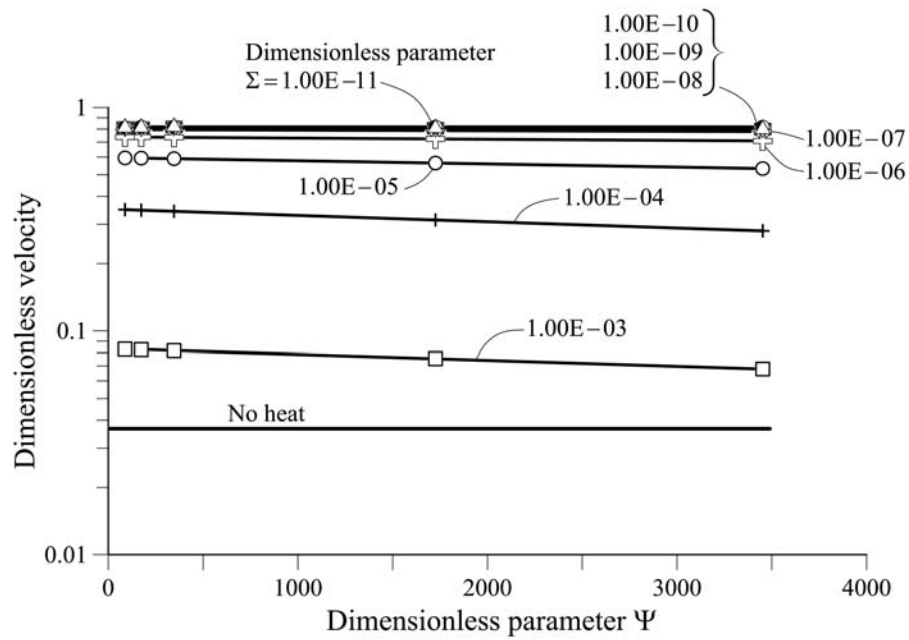
Figure 3. Geometry of a planar landslide and shear band



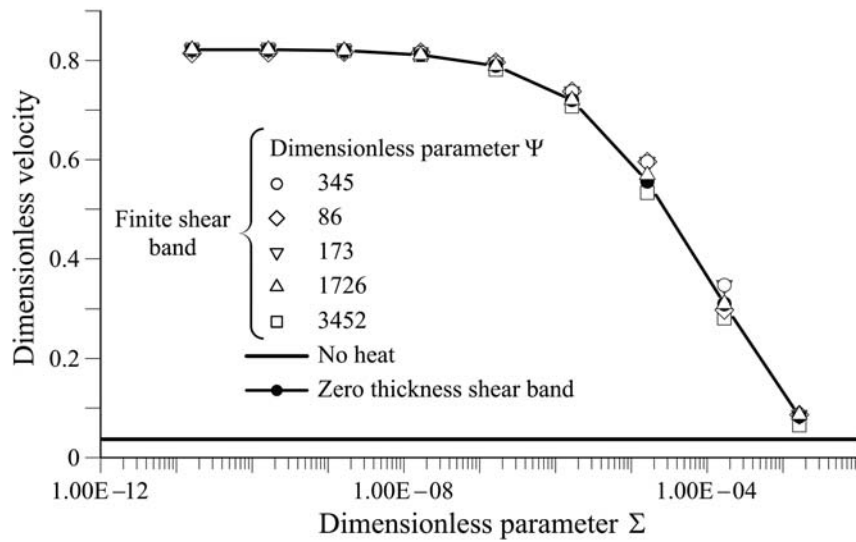


1081 Figure 4. Calculated response of an impending planar slide to an imposed variation of water level (a). (b),
 1082 (c): Temperature at the center of shear band; (d): Excess pore water pressure at the center of shear
 1083 band; (e): slide velocity; (f): slide displacement; (g): evolution of friction angle.

1084

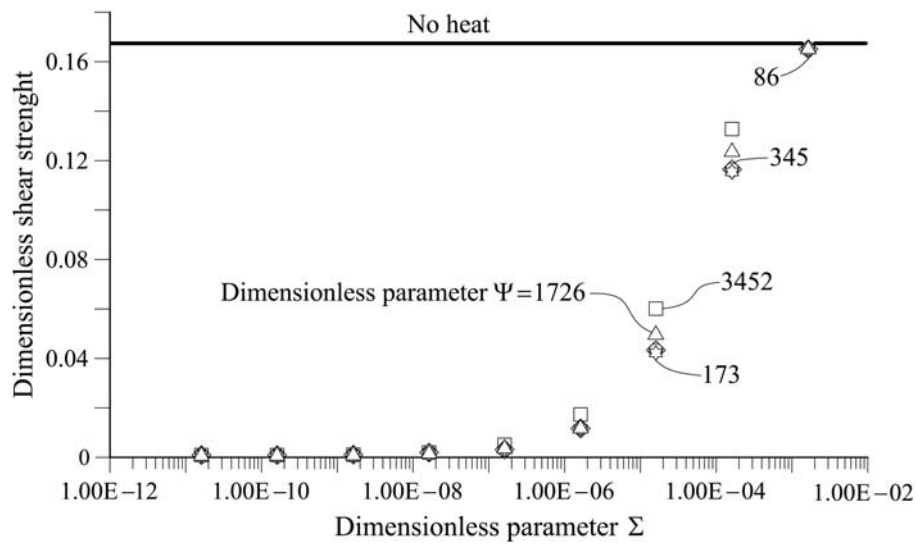


(a)



(b)

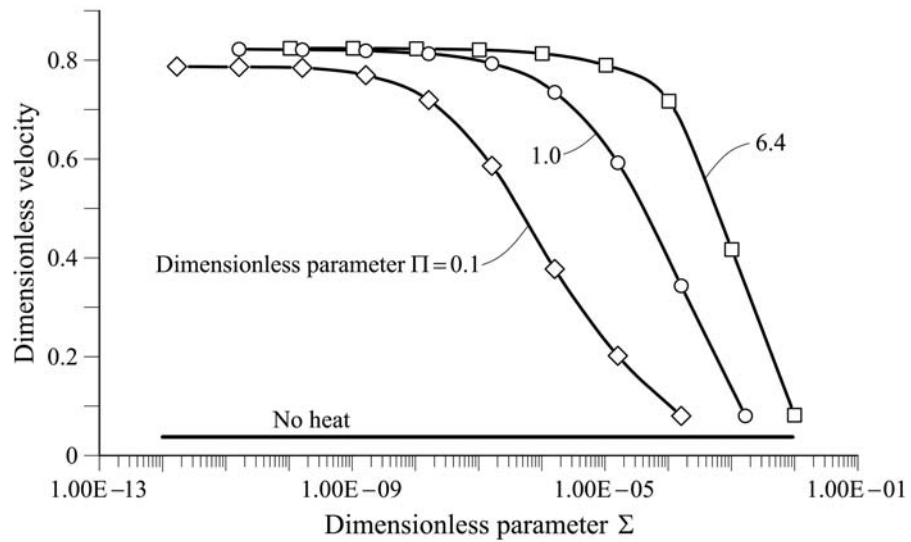
Figure 5. Dimensionless velocity reached at a displacement equal to twice the height of the sliding mass for different values of two dimensionless parameters: parameters Ψ (coefficient of heat source term in the energy balance equation) and Σ , (coefficient of consolidation in the mass balance equation of water flow). Π (coefficient of rate of temperature in the mass balance equation) is equal to 1. Rate independent friction.



1093

1094 Figure 6. Ratio of final and initial shear strength for different values of two dimensionless parameters:
 1095 parameter Ψ (coefficient of heat source term in the energy balance equation) and Σ (coefficient of
 1096 consolidation in the mass balance equation of water flow). Π (coefficient of rate of temperature change
 1097 in the mass balance equation) is equal to 1. Rate independent friction.

1098

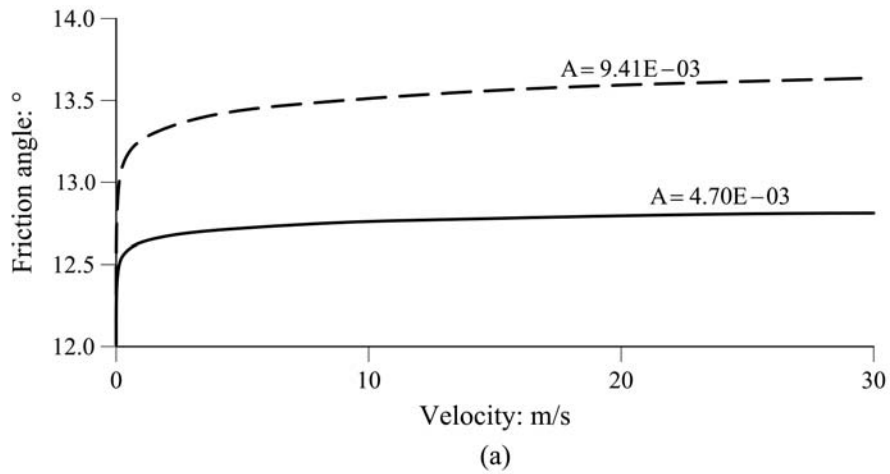


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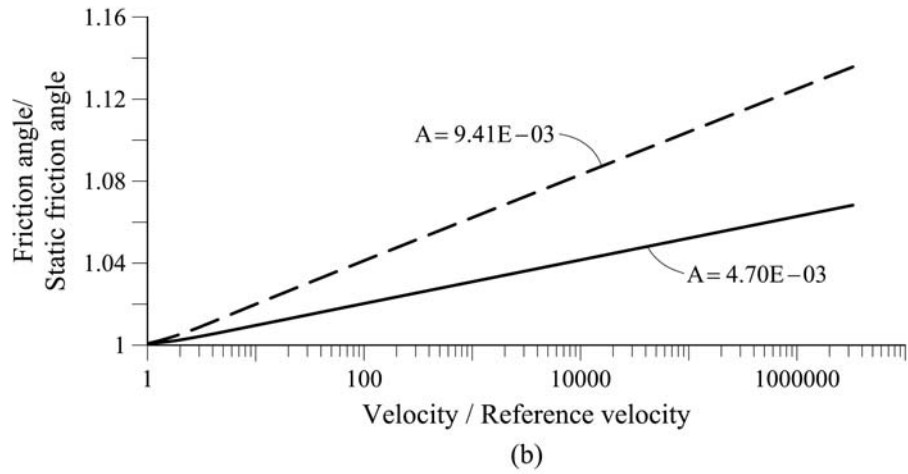
1100 Figure 7. Dimensionless velocity reached at 50 m of displacement for different values of two
 1101 dimensionless parameters: parameter Π (coefficient of rate of temperature change in the mass balance
 1102 equation) and Σ (coefficient of consolidation in the mass balance equation of water flow). Ψ , the
 1103 coefficient of heat in the energy balance equation, is equal to 345. Rate independent friction.

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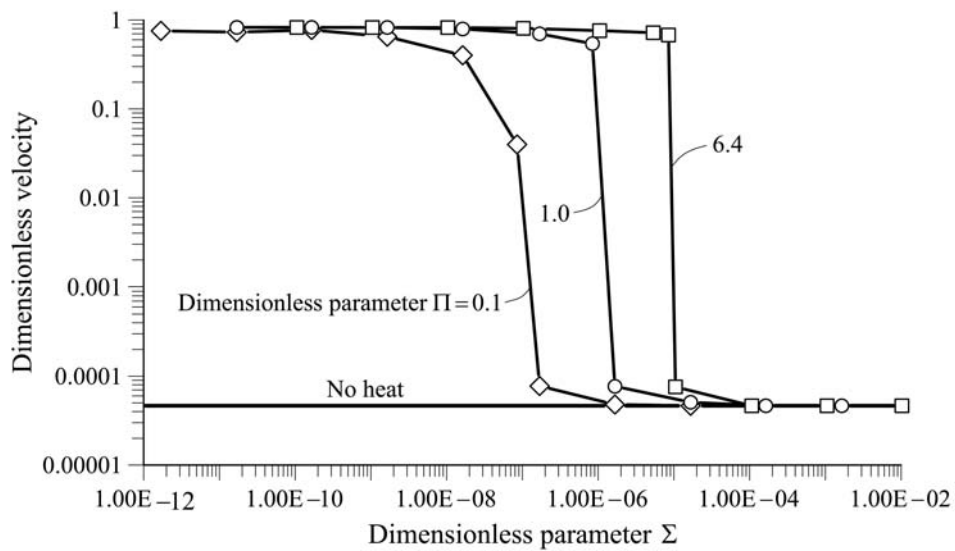


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1107

1108 Figure 8. Friction angle as a function of shearing velocity for two different A parameters. (a) Friction
 1109 angle vs velocity; (b) Ratio between friction angle and static friction angle associated with log velocity
 1110 values lower than a reference vel ($\hat{v}_{ref} = 6.4 \cdot 10^{-7}$).



1111

Figure 9. Dimensionless maximum velocity in terms of dimensionless parameters Π (coefficient of rate of temperature change in the mass balance equation) and Σ (coefficient of consolidation in the mass balance equation of water flow). Ψ (coefficient of heat source term in the energy balance equation) is equal to 345. Rate dependent friction parameters: $A=4.7 \cdot 10^{-3}$ and $\hat{v}_{ref} = 6.4 \cdot 10^{-7}$.

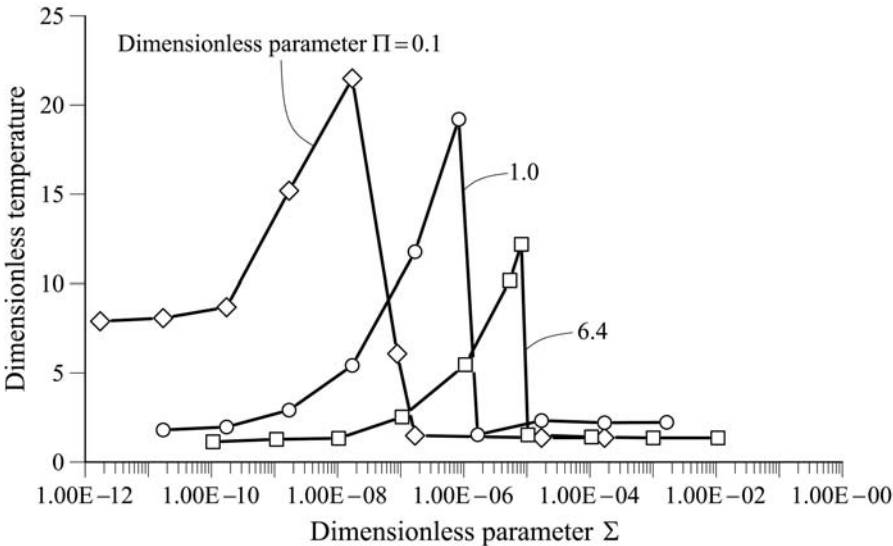


Figure 10. Dimensionless temperature for different dimensionless parameters: parameter Π (coefficient of rate of temperature change in the mass balance equation) and Σ (coefficient of consolidation in the mass balance equation of water flow). Ψ (coefficient of heat source term in the energy balance equation) is equal to 345. Rate dependent friction with $A=4.7 \cdot 10^{-3}$ and a reference velocity equal to $\hat{v}_{ref} = 6.4 \cdot 10^{-7}$.

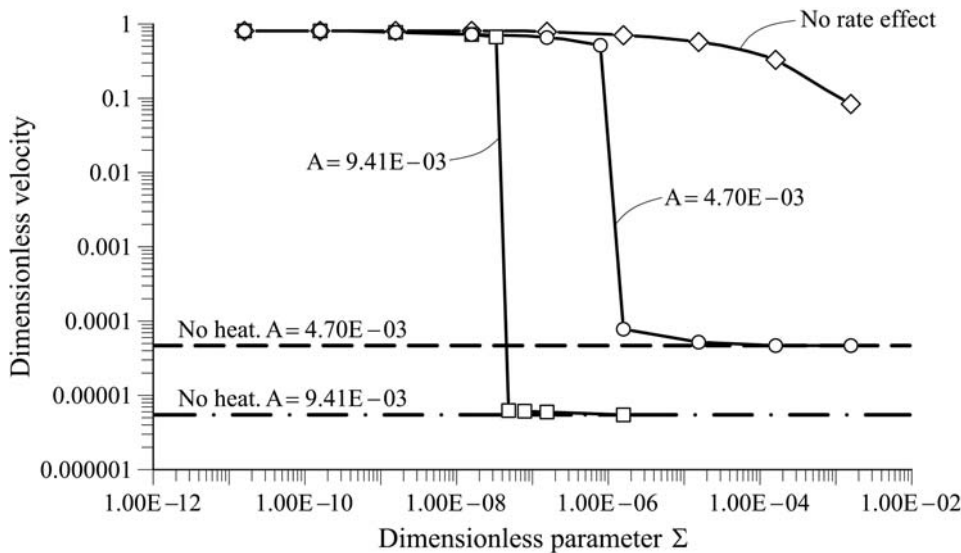
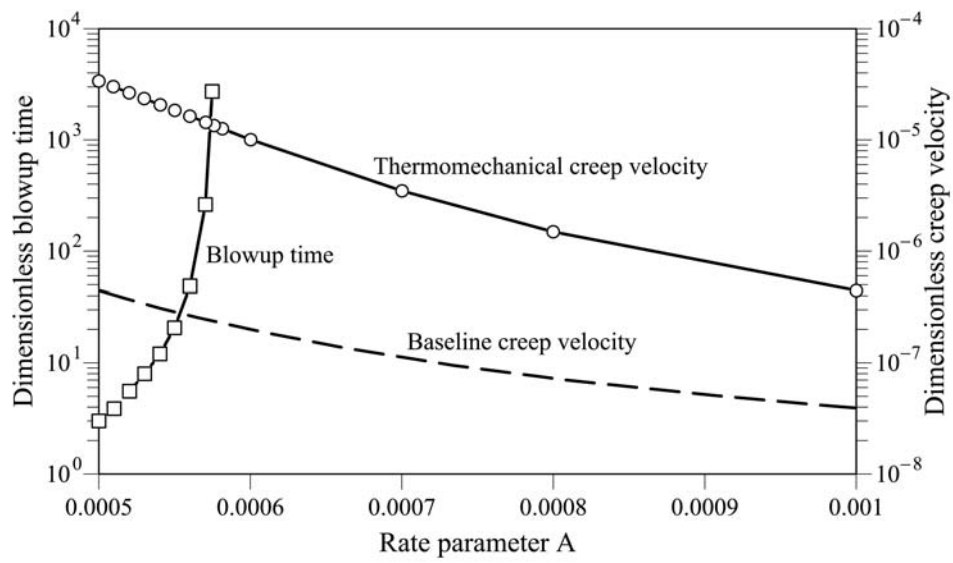
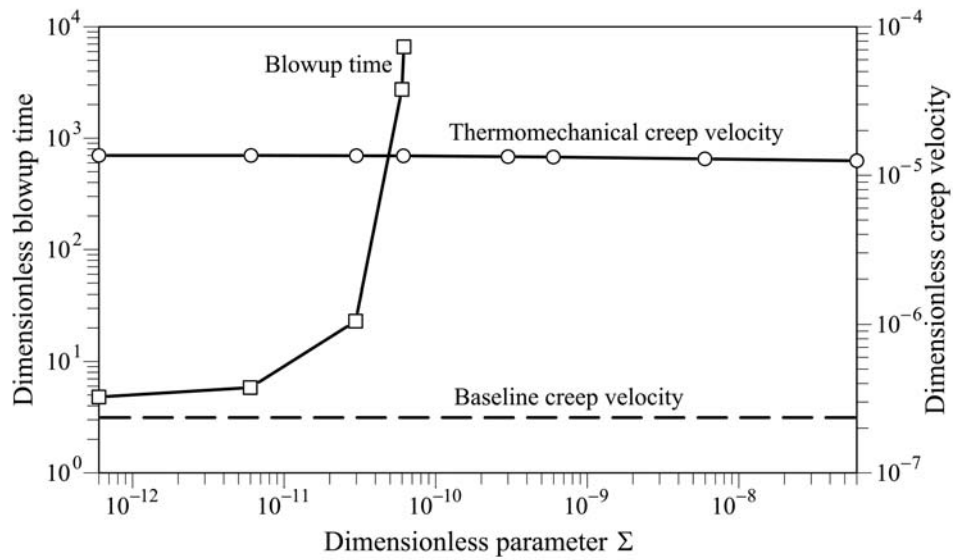


Figure 11. Dimensionless velocity in terms of dimensionless parameters Π (coefficient of rate of temperature change in the mass balance equation) and Σ (coefficient of consolidation in the mass balance equation of water flow) for three values of the rate effect parameter A . Ψ (coefficient of heat source term in the energy balance equation) is equal to 345. In all cases plotted, $v_{ref} = 10^{-5}$ m/s.



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1129 Figure 12. Effect of rate parameter on creep velocity and blowup time



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1131 Figure 13. Effect of the dimensionless parameter Σ (coefficient of consolidation in the mass

1132 balance equation of water flow) on creep velocity and blowup time

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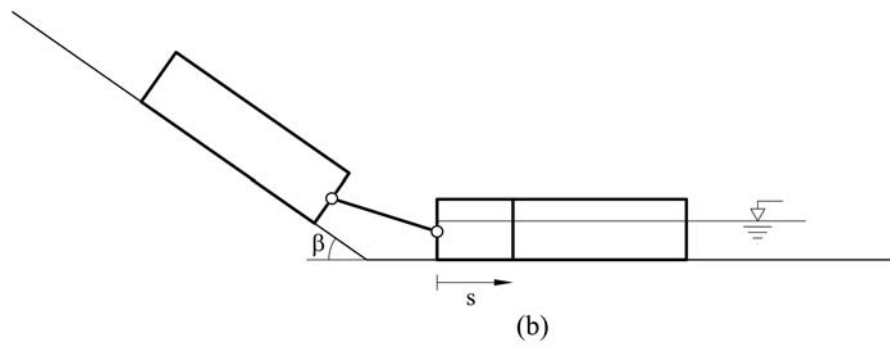
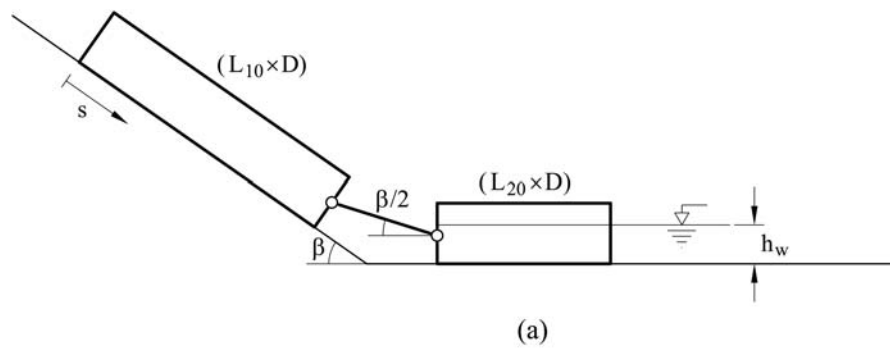
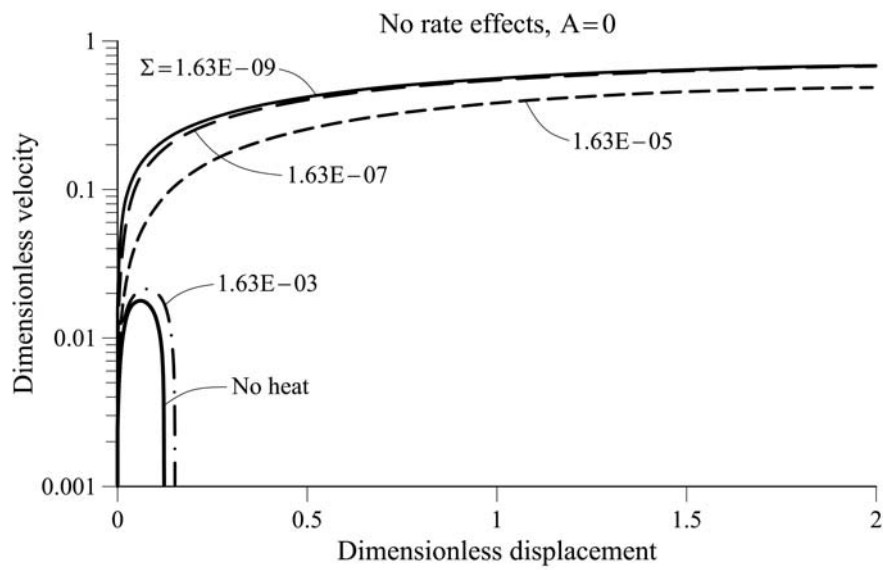


Figure 14. The two-block model analysed.

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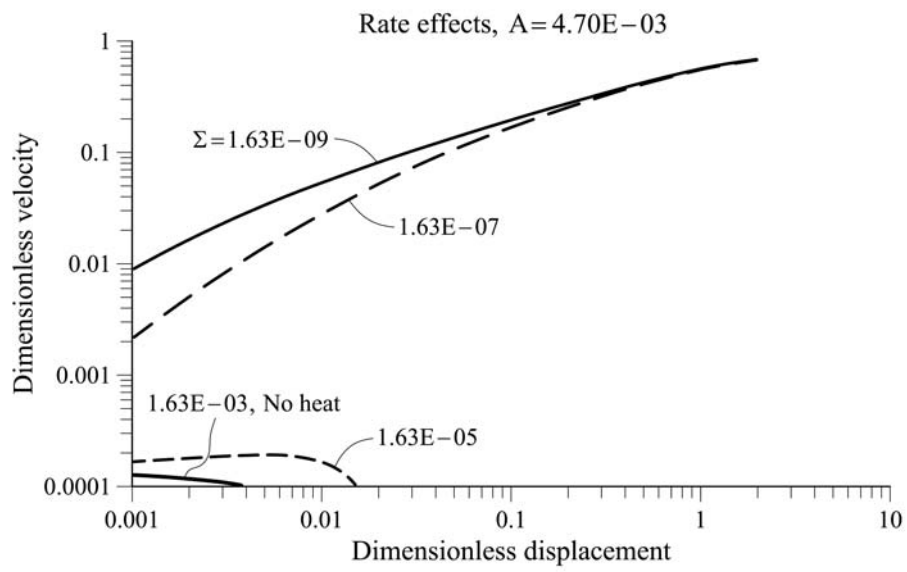
1140 Figure 15. Thermal analysis of two block model; no rate effect on friction ($A=0$). Effect of
 1141 dimensionless parameter Σ (coefficient of consolidation in the mass balance equation of water
 1142 flow) on velocity and run-out.

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1148 Figure 16. Thermal analysis of two block model; rate effect on friction characterized by
 1149 $A=4,7 \cdot 10^{-3}$ and a reference velocity equal to $\hat{v}_{ref} = 6.4 \cdot 10^{-7}$. Effect of band permeability on
 1150 velocity and run-out.

1151

TABLE

Table 1. Contributions to thermal analysis of catastrophic landslides

References	Problem description and governing equations	Sliding geometry	Constitutive modelling	Case analyzed and particular characteristics	Main emphasis
Habib (1967)	Heat induced by friction on a shear band, including heat losses.	Planar	N/A		Generated vapour pressure in the shearing band may explain a rapid motion
Romero and Molina (1974)	Dynamics of motion expressed as a balance of energy: slide deformation, basal friction, impulse of reservoir water and kinetic energy. Pore pressure determined in a phase diagram vapour-liquid water	Compound failure surface. Defined by vertical slices	Mohr-Coulomb	Vaiont. Run-out and velocity determined during the entire motion	Explaining the high velocity of Vaiont
Voigt and Faust (1982)	Rigid mass sliding and deformation concentration in a shear band Saturated shear band Terzaghi's effective stress Mass, energy and momentum balances One dimensional model for heat and pore water pressure dissipation.	Planar	Mohr-Coulomb	Vaiont. Average block. 1D Finite Element analysis	Heat induced fluid pressure explains Vaiont velocity
Vardoulakis (2000, 2002)	As in V&F(1982)	Slip circle	Mohr-Coulomb frictional strength	Vaiont.	Dynamic post failure response

	Numerical analysis		Shear strain and shear strain rate	Slide triggering by reducing	
	Heat advective term negligible		softening of frictional coefficient	the frictional angle from the	
	No vapour generation		Thermal expansion coefficient of the	strict equilibrium	
			soil skeleton including thermoelastic		
			expansion and thermoplastic collapse		
Veveakis et al. (2007)	As in V&F(1982)	Planar	Mohr-Coulomb frictional strength	Vaiont.	Long creeping phase of Vaiont
	Analytical solutions		Thermal softening and velocity	Slide triggering by reducing	landslide and the final high
			strengthening law for frictional	frictional angle from strict	acceleration due to the onset of
			coefficient	equilibrium	thermal pressurization
			Thermal expansion coefficient of the	Constant water table in	
			soil skeleton including thermoelastic	Veveakis et al. (2007)	
			expansion and thermoplastic collapse		
Goren & Aharonov (2007)	As in V&F(1982)	Planar	Mohr-Coulomb frictional strength	Synthetic cases	Long travel distances of
	Numerical analysis		Constant frictional coefficient	Slide triggering by an initial	landslides and their relation to
	Advective term and heat conduction negligible			velocity	slide volumes depending on the
	No vapour generation			Sensitivity analysis	depth dependent permeability
Goren & Aharonov (2009)	As in V&F(1982)	Planar	Mohr-Coulomb frictional strength	Synthetic cases	Sensitivity analysis of the
	Numerical analysis		Constant frictional coefficient	Slide triggering by imposing	triggering factors and
	Advective term and heat conduction negligible		Constant thermal expansion coefficient	an initial velocity (e.g. constitutive and geometrical	parameters to evaluate the
	No vapour generation			simulating an earthquake)	effect on the regime of the
				and imposing an excess pore	

				water pressure that dissipates in time (e.g. simulating rain of snow melt infiltration).	landslide which may become an arrested or catastrophic slide.
Pinyol and Alonso (2010a)	As in V&F(1982)	Planar	Mohr-Coulomb frictional strength	Synthetic cases	Analytical solution for the slide
	Analytical solution		Constant frictional coefficient	Slide triggering by increasing	velocity.
	Advective term and heat conduction negligible		Constant thermal expansion coefficient	the water level	Sensitivity analysis of
	Incompressible water and soil skeleton				constitutive and geometrical
	No vapour generation				parameters to evaluate the slide evolution.
Pinyol and Alonso (2010b)	As in V&F(1982)	Compound failure	Mohr-Coulomb frictional strength	Vaoint.	Dynamic post failure response.
	Numerical analysis	surface. Two	Constant frictional coefficient	Slide triggering by increasing	Sensitivity analysis of
	Advective term and heat conduction negligible	interacting	Constant thermal expansion coefficient	the water reservoir level	constitutive parameters and
	No vapour generation	wedges describe evolving geometry		from the strict equilibrium	scale effects.
Goren et al (2010)	As in V&F(1982)	Planar sliding	Mohr-Coulomb frictional strength	The Heart Mountain	Simulation of the long runout
	Numerical analysis	travelling along a	Constant frictional coefficient	landslide in Canada.	features of the simulated
	Advective term and heat conduction negligible	surface not totally	Constant thermal expansion coefficient		landslides
	No vapour generation	planar	Thermal decomposition of Dolomite which involve increments in porosity		

Non constant permeability					
Cecinato et al. (2011)		Planar and slip	Thermoplastic Modified Cam clay	Vaiont in Cecinato et al.	Evaluation the effect of the
	As in V&F(1982)	circle	Strain, strain rate and temperature	(2011)	thermal, strain and strain rate
Cecinato and Zervos (2012)	Numerical analysis		softening of the critical state parameter		softening and the pressurization
	Heat advective term negligible		function of the friction angle at critical	Jiufenghershans landslide	mechanism in Cecinato et al.
			states	(Taiwan) in Cecinato and	(2011)
			Thermal expansion coefficient of the	Zervos (2012)	
			soil skeleton including thermoelastic		Parametric analysis in Cecinato
			expansion and thermoplastic collapse		and Zervos (2012)
He et al. (2015)	As in V&F(1982)	Planar	Mohr-Coulomb frictional strength	Synthetic case	A depth-averaged model
	Numerical analysis		Constant frictional coefficient		coupled with a thermo-poro-
	Advective term and heat conduction negligible		Constant thermal expansion coefficient		elastic approach in the shear
	Depth-average integration model				zone

Table 2. Material properties for water and solid particles

<i>Parameter</i>	<i>Symbol</i>	<i>Value</i>	<i>Unit</i>
<i>Water</i>			
Density	ρ_w	1,000	kg/m ³
Coefficient of compressibility	α_w	5×10^{-10}	1/Pa
Thermal expansion coefficient	β_w	3.42×10^{-4}	1/°C
Specific heat	c_w	4.186×10^3	J/kg·°C
Conductivity	Γ	0.580	J/m·s·°C
<i>Solid particles</i>			
Density	ρ_s	2,700	kg/m ³
Thermal expansion coefficient	β_s	3×10^{-5}	1/°C
Specific heat	c_s	8.372×10^{-2}	J/kg·°C
Conductivity	Γ	0.375	J/m·s·°C
<i>Soil</i>			
Porosity	n	0.2	-

Table 3. Typical ranges permeability, coefficient of compressibility and band thickness

<i>Parameter</i>	<i>Symbol</i>	<i>Range of value</i>	<i>Unit</i>
Permeability	k	$10^{-13} - 10^{-5}$	m/s
Coefficient of compressibility	m_v	$10^{-10} - 10^{-8}$	1/Pa

Shear band thickness	$2e$	0.001-0.05	M
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